LRFD

Section 1.2

New: January 2005

CLICK HERE TO Index

Index

1.2.1 Permanent Loads

1.1 Dead Load

LRFD 3.5.1

Included in dead loads are the weights of all structural components, appurtenances and utilities attached, wearing surfaces, earth cover, future overlays, and planned widenings. The following are the LRFD code designations:

- *DC* – dead load of structural components and nonstructural attachments
- *DW* – dead load of wearing surfaces and utilities
- *EV* – vertical pressure from dead load of earth fill

Table 1.2.1.1 Unit Weights of Dead Loads

Aluminum Alloys		0.175 kcf
Bituminous Wearing Surfaces		0.140 kcf
*Plain Concrete	Lightweight	0.110 kcf
	Sand-Lightweight	0.120 kcf
	Normal Weight w/ f'c ≤ 5.0 ksi	0.145 kcf
	Normal Weight w/ 5.0 < f'c ≤ 15 ksi	0.140 kcf + .001 f'c
Compacted Sand, Silt, or Clay		0.120 kcf
Loose Sand, Silt, or Gravel		0.100 kcf
Soft Clay		0.100 kcf
Steel		0.490 kcf
Stone Masonry		0.170 kcf
Water	Fresh	0.0624 kcf
	Salt	0.0640 kcf

* Note: Add 0.005 kcf for Reinforced Concrete.

1.2 Earth Load

LRFD 3.5.2, 3.11

Earth pressure is considered a function of type and unit weight of earth, water content, soil creep characteristics, degree of compaction, location of groundwater table, earth-structure interaction, amount of surcharge, and earthquake effects. The following are the LRFD designations for the earth loads:

- *EH* – horizontal earth pressure load
- *ES* – earth surcharge load
- *DD* – downdrag

Earth Pressure Load, EH
Basic earth pressure is assumed to be linearly proportional to depth and is taken as:

LRFD 3.11.5.1

$$p = k\gamma_s z$$

Where:
p = basic earth pressure, ksf
k = coefficient of lateral earth pressure taken as k_o, as specified in LRFD 3.11.5.2 for walls that do not deflect or move, or k_a for walls that deflect or move sufficiently to reach minimum active conditions, as specified in LRFD 3.11.5.3, 3.11.5.6, and 3.11.5.7
γ_s = unit weight of soil, kcf
z = depth below the surface of earth, ft.

Passive earth pressure can be estimated for cohesive soils by the following equation:

LRFD 3.11.5.4

$$p_p = k_p\gamma_s Z + 2c\sqrt{k_p}$$

Where:
p_p = passive earth pressure, ksf
k_p = coefficient of passive pressure as specified in LRFD Figures 3.11.5.4.-1 and 3.11.5.4-2
γ_s = unit weight of soil, kcf
Z = depth below surface of soil, ft.
c = unit cohesion, ksf

Note: Passive pressure shall be considered a Resistance and shall not be considered a Loading.

LRFD 3.11.5.6

Earth Pressures for Anchored Walls
For anchored walls with only one level of anchors, the earth pressure may be assumed to be linearly proportional to depth. For anchored walls with two or more levels of anchors, the earth pressure may be assumed constant with depth.

LRFD 3.11.5.8

Earth Pressures for Mechanically Stabilized Earth (MSE) Walls

The resultant force per unit width behind an MSE wall, as acting at a height of h/3 above the base of the wall and parallel to the slope of the backfill, shall be:

$$P_a = \frac{1}{2}\gamma_s h^2 k_a$$

Where:

P_a = force resultant per unit width, kip/ft.
k_a = coefficient of active earth pressure as specified in LRFD 3.11.5.3
γ_s = unit weight of soil, kcf
h = notional height of horizontal earth pressure diagram

Earth Surcharge Load, ES

A constant horizontal earth pressure shall be added to the basic earth pressure when there is a uniform surcharge present. This constant earth pressure is calculated by the following equation:

LRFD 3.11.6.1

$$\Delta_p = k_s q_s$$

Where:

Δ_p = horizontal earth pressure due to uniform surcharge, ksf
k_s = coefficient of earth pressure due to surcharge
q_s = uniform surcharge applied to the upper surface of the active earth wedge, ksf

Live Load Surcharge, LS

LRFD 3.11.6.4

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to the wall height behind the back face of the wall.

$$\Delta_p = k\gamma_s h_{eq}$$

Where:

Δ_p = uniform earth pressure due to live load surcharge, ksf
k = coefficient of earth pressure
γ_s = unit weight of soil, kcf
h_{eq} = equivalent height of soil for the design truck, ft.

Equivalent heights of soil, h_{eq}, may be taken from the following tables. Use Table 1.2.1.2 for abutments and generally any wall running perpendicular to the direction of traffic. Use Table 1.2.1.3 for retaining walls or any wall running parallel to the direction of traffic. The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing. Linear interpolation may be used for intermediate wall heights.

LRFD Table 3.11.6.4-1

Table 1.2.1.2 Equivalent Height of Soil - Abutment

Wall Height, ft.	h_{eq}, ft.
5	4.0
10	3.0
20 or higher	2.0

LRFD Table 3.11.6.4-2

Table 1.2.1.3 Equivalent Height of Soil - Retaining Walls

Wall Height, ft.	Distance from back face of wall to the wheel line	
	0.0 ft.	1.0 ft. or further
5	5.0	2.0
10	3.5	2.0
20 or higher	2.0	2.0

The distance from the back face of wall to edge of traveled way of 0.0 ft. corresponds to placement of a point wheel load 2.0 ft. from the back face of the wall. For the case of the uniformly distributed lane load, the 0.0 ft. distance corresponds to the edge of the 10.0 ft. wide traffic lane.

1.2.2 Transient Loads

2.1 Live Load

Vehicular Live Load, LL

LRFD 3.6.1.2

The design vehicular live load HL-93 shall be used. It consists of either the design truck or a combination of design truck and design lane load.

The number of design lanes shall be calculated by taking the integer part of the ratio of the clear roadway width in feet divided by 10.0 ft. In cases where the traffic lane width is less than 10.0 ft. wide, the width of the design lane shall be equal to the traffic lane width.

The extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor to account for the probability of simultaneous lane occupation of the design truck. The following table gives the multiple presence factors, *m*.

Table 1.2.2.1 Multiple Presence Factors, *m*.

Number of Loaded Lanes	Multiple Presence Factor, *m*
1	1.20
2	1.00
3	0.85
4 or more	0.65

Multiple presence factors are not to be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. Thus, the factor 1.20 must be removed from the single lane distribution factors when they are used to investigate fatigue.

For slab design, where the approximate strip method is used, the force effects shall be determined on the following basis:

- Where primary strips are transverse and their span does not exceed 15 ft., the design shall be based on the axle loads of the design truck or tandem alone.

- Where primary strips are transverse and their span exceeds 15 ft., the design shall be based on the axle and lane loads together.

Design Truck

Figure 1 and Figure 2 describe the design truck load. Dynamic load allowance should be considered.

Design Tandem

A design tandem shall consist of a pair of 25.0 kip axles spaced 4.0 ft. apart longitudinally and spaced 6.0 ft. transversely. The design tandem should be considered with dynamic allowance.

Design Lane Load
A load of 0.64 klf, uniformly distributed in the longitudinal direction shall be the design lane load. The design lane load also shall be uniformly distributed transversely over a 10.0 ft. width. Design lane load should not be considered with dynamic allowance.

LRFD 3.6.1.6

Pedestrian Load, PL
Pedestrian live load on sidewalks greater than 2 ft. wide shall be:
PL = 0.075 ksf

This does not include bridges designed exclusively for pedestrians or bicycles. Due to possible future roadway widening, sidewalks as shown below shall be designed assuming roadway loadings.

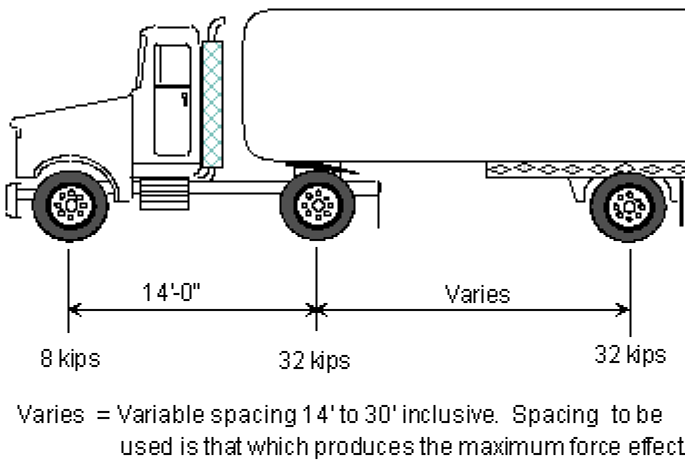

Figure 1.2.2.1 Design Truck

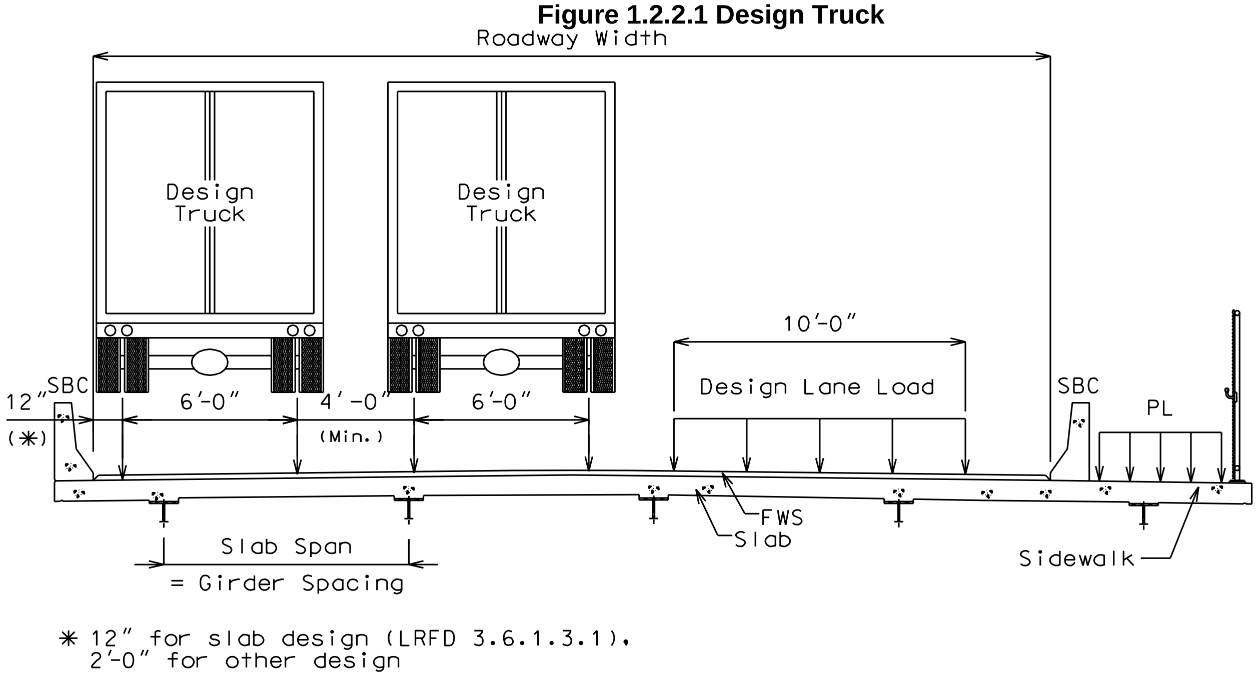

Figure 1.2.2.2 Summary of Live Loads

LRFD 3.6.2

Dynamic Load Allowance, IM

The dynamic load allowance shall be applied to Design Truck or Tandem loading only. The dynamic load allowance replaces the effect of impact used in AASHTO Standard Specifications. It accounts for wheel load impact from moving vehicles. The static effect of the vehicle live load shall be increased by the percentage specified in Table 1.2.2.2.

LRFD Table 3.6.2.1-1

Table 1.2.2.2 Dynamic Load Allowance, *IM*

Components	*IM*
Deck Joints – All Limit States	75%
Other Components – Fatigue and Fracture Limit State	15%
All Other Limit States	33%

The factor to be applied to the static load shall be taken as: $\left(1+IM\right)$

The dynamic load allowance is not to be applied to pedestrian or design lane loads, retaining walls not subject to vertical reactions from the superstructure, and foundation components that are entirely below ground level.

LRFD 3.6.1.3.1

HL-93 Live Load Application

Extreme force effects shall be taken as the larger of:

- Design Tandem + Design Lane
- Design Truck + Design Lane

For negative moment between points of contraflexure and for reactions at intermediate bents the following shall also be considered:

- 90% of two design trucks spaced a minimum of 50.0 ft. including the design lane

Live Load Position

Axles that do not contribute to the maximum force effect shall not be included.

For designing the deck overhang, the center of a truck wheel shall not be closer than 1 ft. from the face of barrier curb.

For all other applications, the center of a truck wheel shall not be closer than 2 ft. from the face of barrier curb.

2.2 Water Load, WA (Major river crossings only)

LRFD 3.3.2

The LRFD load designation for water load and stream pressure is *WA*.

LRFD 3.7.1

Concerning static pressure of water, the load will be assumed to act perpendicular to the surface that is retaining the water. The pressure is calculated as the product of height of water above the point of consideration and the specific weight of water.

LRFD 3.7.2

Buoyancy is considered to be an uplift force, taken as the sum of the vertical components of static pressures acting on all components below the design water level.

LRFD 3.7.3.1

Stream Pressure – Longitudinal

The stream pressure acting in the longitudinal direction of substructures is given by the following equation

$$p = \frac{C_D V^2}{1000}$$

Where:

p = pressure of flowing water, ksf

C_D = drag coefficient for piers as specified in LRFD Table 3.7.3.1-1

V = design velocity of water for the design flood in strength and service limit states and for the flood check in the Extreme Event limit state, fps *

LRFD 3.7.3.2

Stream Pressure – Lateral

The uniformly distributed stream pressure acting in the lateral direction of substructures due to water flowing at an angle, θ, to the longitudinal axis of the pier is given by the following equation

$$p = \frac{C_L V^2}{1000}$$

Where:

p = lateral pressure, ksf

C_L = lateral drag coefficient specified in LRFD Table 3.7.3.2-1

V = design velocity of water for the design flood in strength and service limit states and for the flood check in the Extreme Event limit state, fps *

* Velocity will be given in the design layout or obtain information from USGS.

2.3 Wind Load

LRFD 3.8.1.1

Wind pressures are calculated from an assumed wind velocity of 100 mph. Wind load shall be uniformly distributed on the area exposed to the wind. The two LRFD load designations for wind loads are WS and WL. Where WS is the wind pressure on structures and WL is the wind pressure on vehicles.

Wind Load to Superstructure, WS

Transverse

LRFD Table 3.8.1.2.2-1

A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure.

- Trusses and Arches = 75 psf
- Girders and Beams = 50 psf ***** (for plate girder lateral bracing check only)

The total wind force shall not be less than 300 pounds per linear foot in the plane of windward chord and 150 pounds per linear foot in the plane of the leeward chord on truss spans, and not less than 300 pounds per linear foot on girder spans.

Wind Load to Substructure from Superstructure, WS

This transverse wind force will be applied at the top of the beam cap for the design of the substructure.

Transverse

A base design wind pressure shall be applied horizontally at right angles to the longitudinal axis of the structure.

- Trusses and Arches, P_B = psf
- Girders and Beams, P_B = psf *****

***** When bridges or parts of bridges are more than 30.0 ft. above low ground or water level, the design wind velocity, V_{DZ}, should be adjusted according to the following equation:

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right)$$

Where:
V_{DZ} = Design wind velocity at design elevation Z, mph
V_{30} = Wind velocity at 30.0 ft. above low ground or above design water level, assume to be 100 mph
V_B = Base wind velocity of 100 mph at 30.0 ft. height.
Z = Height of structure at which wind loads are being calculated as measured from low ground or from water level, ft.
V_0 = Friction velocity as specified in Table 1.2.2.3, mph
Z_0 = Friction length of upstream fetch as specified in Table 1.2.2.3, ft.

The corresponding design wind pressure shall be calculated from the following equation:

$$P_D = P_B \frac{V_{DZ}^{\ 2}}{10{,}000} \geq P_B$$

Where:
P_D = Design wind pressure, ksf

LRFD Table 3.8.1.1-1

Table 1.2.2.3 V_0 and Z_0 for Various Upstream Surface Conditions

Condition	Open Country	Suburban	City
V_0, mph	8.2	10.9	12.0
Z_0, ft.	0.23	3.28	8.20

Longitudinal **
The standard wind force in the longitudinal direction shall be applied as the following:

- Truss and Arch Structures *W* = 20 psf
- Girder Structures *W* = 12 psf

The total longitudinal wind force, *P,* will be:

P = *L H W*

Where:
L = the overall bridge length, ft.
H = the total height of the girders, slab, barrier curb and any superelevation of the roadway, ft.
W = wind force per unit area, psf

****** This longitudinal force is distributed to the bents based on their stiffness. The longitudinal wind force for the bent will be applied at the top of the beam cap for the design of the substructure.

Wind Load Applied Directly to the Substructure, WS

LRFD 3.8.1.2.3

The parallel and perpendicular forces to be applied directly to the substructure elements shall be calculated from an assumed base wind pressure of 40 psf. This wind force per unit area shall be multiplied by the exposed area of each substructure member in elevation (use front view for perpendicular force and side view for parallel force, respectively). These forces are acting at the center of gravity of the exposed portion of the member.

A shape factor of 0.7 shall be used in applying wind forces to round substructure members.

Wind Pressure on Vehicles, WL

LRFD 3.8.1.3

A force of 100 pounds per linear foot of the structure shall be applied transversely to the structure along with a force of 40 pounds per linear foot longitudinally. These forces are assumed to act 6 feet above the top of slab.

The transverse force is applied at the bents based on the length of the adjacent spans affecting them. The longitudinal force is distributed to the bents based on their stiffness.

2.4 Superimposed Deformations

Internal forces due to creep, shrinkage, and temperature ranges shall be considered. The corresponding LRFD load designations are:

LRFD 3.3.2

TU = uniform temperature
CR = creep
SH = shrinkage

Uniform Temperature, TU

LRFD 3.12.2

Temperature stresses on steel and concrete structures shall be calculated from the following temperature ranges:

Table 1.2.2.4 Temperature Ranges

	Temperature Rise	**Temperature Fall**	**Temperature Range**
Steel	60° F	90° F	150° F
Concrete	50° F	70° F	120° F

Note: Ranges are from a construction temperature of 60° F.

Creep, CR

LRFD 3.12.5

Creep strains shall be calculated in accordance with LRFD 5.4.2.3 Dependence on time and changes in compressive stresses shall be taken into account.

Shrinkage, SH

LRFD 3.12.4

Differential shrinkage strains between concretes of different age and composition and between concrete and steel shall be determined in accordance with the provisions of LRFD 5.4.2.3

2.5 Other Loads

Braking Force, BR

LRFD 3.6.4

The Braking Force shall be the greater of:

- 25% of the axle weights of the design truck or tandem per lane or
- 5% of the design reaction

Braking forces shall be placed in all design lanes carrying traffic in the same direction with a multiple presence factor applied. These forces shall be assumed to act horizontally at a distance of 6.0 ft. above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

Centrifugal Force, CE

LRFD 3.6.3

Structures on curves shall be designed for a horizontal radial force equal to the following percentage of axle weights of the design truck or tandem.

$$C = \frac{4 * v^2}{3 * g * R}$$

Where:
C = Percentage of axle weights of the design truck or tandem
v = Highway design speed, ft./sec.
g = Acceleration of gravity: 32.2 ft./sec.^{2}
R = Radius of curvature of traffic lane, ft.

This force shall be applied horizontally at a distance of 6.0 ft. above the roadway surface. Multiple presence factors shall apply.

Ice Load, IC

Ice forces should only be included when noted on the Design Layout. The LRFD load designation for ice is *IC* and is discussed in LRFD Section 3.9.

Earthquake, EQ

Bridge Manual Section 6.1 Seismic Design for earthquake load design.

LRFD 3.6.5

Vehicular Collision Force, CT

Abutments and piers located within a distance of 30 feet to the edge of roadway or within a distance of 50 feet to the centerline of a railway track, shall be designed for an equivalent static force of 400 kip, which is assumed to act in any direction in a horizontal plane, at a distance of 4 feet above ground.

Design for vehicular collision force of 400 kip is not required if abutment or pier is protected by:

- An embankment
- A 54 inch TL5 barrier located within 10 feet from the pier or abutment.
- A 42 inch TL5 barrier located more than 10 feet from the pier or abutment
- A collision wall meeting the standards provided in these guidelines. See LRFD DG Sec. 3.71.
- A guardrail or barrier that is consistent with the department's roadway standards. See LRFD DG Sec. 3.32.

Note: TL5 refers to a "Test Level Five" crash test. See LRFD 13.7.2 for more specifics.

1.2.3 Load Factors and Combinations

3.1 Load Factors

LRFD 3.4.1

The total factored force effect shall be given by the following equation:

$Q = \Sigma\eta_i\gamma_i Q_i$

Where:

η = load modifier specified in LRFD 1.3.2 (η = 1.0 shall be used unless directed otherwise by the Structural Project Manager)
Q_i = force effects from loads
γ_i = load factors

All of the load factors for the various load combinations are specified in LRFD Table 3.4.1-1 and Table 3.4.1-2. The load factors shall be selected to produce total extreme factored force effects, both positive and negative extremes should be checked. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect.

For permanent force effects, the load factor that produces the more critical combination shall be selected from LRFD Table 3.4.1-2. Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated.

3.2 Load Combinations

Strength I

LRFD 3.4.1

This is the basic loading combination pertaining to normal vehicular use of the structure without the effects of wind.

Strength II

Combination of loads relating to owner specified special design vehicles, evaluation permit vehicles, or both without the effects of wind.

Strength III

Combination for structures exposed to a wind velocity exceeding 55 mph. Vehicles become unstable at higher wind velocities preventing the presence of significant live load.

Strength IV

Combination relating to very high dead load to live load force effect ratios. Strength IV is more likely to control on bridges with very large spans where the dead load to live load ratio exceeds about 7.0.

Strength V

Combination relating to normal vehicular use of a structure with a wind velocity of 55 mph.

Extreme Event I

Combination including earthquake.

Extreme Event II

Combination relating to ice loads, collision by vehicles or vessels, and certain hydraulic events with reduced live load other than that which is part of the vehicular collision load, *CT*. Since the joint probability of these events is very low, events are to be investigated one at a time.

Service I

Combination relating to normal vehicular use of a structure with a wind velocity of 55 mph. This load combination is used to control crack width in reinforced concrete structures, to check compression in prestressed concrete components, and to investigate slope stability.

Service II

Combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

Service III

Combination only relating to tension in prestressed concrete structures with the objective of crack control.

Service IV

Load combination relating only to tension in prestress concrete substructures with the objective of crack control.

Fatigue

Fatigue and fracture load combination relating to repetitive vehicular live load and dynamic responses under a single design truck as specified in LRFD 3.6.1.4.1.

1.2.4 Load Distributions

4.1 Live Load Moment Distribution

LRFD Table 4.6.2.2.2b-1

The following equations give the moment distribution factors of interior girders for concrete deck bridges with Prestressed-I or Steel Plate girders/stringers. The larger value obtained from these equations shall be the distribution factor used. Note that the Multiple Presence Factors are included in the equations.

Distribution to Interior Girder - One Design Lane Loaded

$$g_{interior} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

Distribution to Interior Girder - Two or More Design Lanes Loaded

$$g_{interior} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

Where:

$\frac{K_g}{12Lt_s^{\,3}}$ May be assumed equal to 1.0 for preliminary design.

S = spacing of beams or webs, ft.
L = span of beam, ft.
t_s = depth of concrete slab, in.
N_b = number of beams or girders

K_g = longitudinal stiffness parameter = $\frac{E_S}{E_D}\left(I + Ae_g^{\,2}\right)$

Where:
E_S = modulus of elasticity of beam material, ksi
E_D = modulus of elasticity of deck material, ksi
I = moment of inertia of beam, $in.^4$
e_g = distance between the center of gravity of the basic beam and center of gravity of deck, in.
A = area of beam, $in.^2$

The above equations may be used if the following conditions are met:

1) 3.5 ft. ≤ S ≤ 16.0 ft.
2) 4.5 in. ≤ t_s ≤ 12.0 in.
3) 20.0 ft. ≤ L ≤ 240.0 ft.
4) $N_b \geq 4$
5) 10,000 ≤ K_g ≤ 7,000,000

Distribution to Exterior Girders

Live load distribution for exterior beams shall be assumed as the larger of the value obtained from the Lever Rule or the following equations:

LRFD Table 4.6.2.2.2d-1

$$g_{exterior} = \left(0.77 + \frac{d_e}{9.1}\right) g_{interior}$$

Where:

d_e = distance from exterior face of the web of exterior beam and the interior edge of curb or traffic barrier, ft.

The above equation may be used if the following condition is met:

1) $-1.0\, ft. \le d_e \le 5.5\, ft.$

LRFD C 4.6.2.2.2d

Additional Provision for Exterior Girder

An additional check is required for exterior girders because the distribution factors for girders in a multigirder cross-section was determined without consideration of diaphragms or cross-frames. The following procedure is the same as the conventional approximation for loads on piles.

$$g_{exterior} = mR$$

Where:

m = multiple presence factor

$$R = \frac{N_L}{N_b} + \frac{X_{ext.} \sum^{N_L} e}{\sum^{N_b} x^2}$$

Where:

N_L = number of lanes investigated

N_b = number of girders

$X_{ext.}$ = horizontal distance from the center of gravity of the pattern of girders to the exterior girder, ft.

e = eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders, ft.

x = horizontal distance from the center of gravity of the pattern of girders to each girder, ft.

Skew Reduction Factor

LRFD Table 4.6.2.2.2e-1

Moment distribution factors would be reduced by the Skew Factor. The Skew Factor will not be used for moment calculations.

4.2 Live Load Deflection

Lateral distribution of live load shall be determined by the following equation. The number of lanes shall be calculated based on 10 ft. lanes.

Live Load Deflection Factor = $\dfrac{N_L}{N_g}$

Where:
N_L = Number of lanes based on 10 ft. lanes
N_g = Number of girder lines

The LRFD HL-93 design load used for calculating deflection shall be the larger of:

- The design truck
- The lane load plus 25% of the design truck

Dynamic load allowance, *IM*, shall be included in the calculations.

Note: By using the above distribution factor without the multiple presence factor, *m*, and by using adjusted allowable deflection values, comparable results to the method used in the MoDOT Bridge Manual guidelines for Load Factor Design for the HS20 design loading are achieved. For bridges previously designed with the HS20-Modified loading, the allowable live load deflection can be modified by 25%. The 25% accounts for the HS20-Mod. Design truck being 25% heavier than the HS20 design truck.

For span lengths over 120 ft., typically the lane loading will control over the truck loading. Therefore, both the allowable deflections corresponding to the HS20 and HS20 Mod. Loadings with span lengths over 120 ft., need to be further adjusted to account for impact being applied to the former LFD lane load whereas under LRFD design the dynamic load allowance is not applied to the lane load.

Table 1.2.4.2.1 Allowable Live Load Deflection

Route Location	NHS Routes & Commercial Zones		All other locations	
Length of Span	L ≤ 120 ft.	L > 120 ft.	L ≤ 120 ft.	L > 120 ft.
HL93 Allowable Deflection	$\dfrac{L}{1250}$	$\dfrac{L}{1000\left(1.25+\dfrac{50}{L+125}\right)}$	$\dfrac{L}{1000}$	$\dfrac{L}{1000\left(1+\dfrac{50}{L+125}\right)}$

Where:
L = Span length distance between centerlines of support, ft.

4.3 Live Load Shear Distribution

LRFD Table 4.6.2.2.3a-1

The following equations give the shear distribution factors of interior girders for concrete deck bridges with Prestressed-I or Steel Plate girders. Note that the Multiple Presence Factors are included in the equations.

Distribution to Interior Girders - One Design Lane Loaded

$$g_{interior} = 0.36 + \left(\frac{S}{25.0}\right)$$

Distribution to Interior Girders - Two or More Design Lanes Loaded

$$g_{interior} = 0.2 + \left(\frac{S}{12.0}\right) - \left(\frac{S}{35.0}\right)^{2.0}$$

The above equations may be used if the following conditions are met:

1) $3.5\,ft. \le S \le 16.0\,ft.$
2) $4.5\ in. \le t_s \le 12.0\ in.$
3) $20.0\,ft. \le L \le 240.0\,ft.$
4) $N_b \ge 4$

Where:
S = spacing of beams or webs, ft.
L = span of beam, ft.
t_s = depth of concrete slab, in.
N_b = number of beams or girders

Distribution to Exterior Girders
Live load distribution for exterior beams shall be assumed as the larger of the value obtained from the Lever Rule or the following equations:

LRFD Table 4.6.2.2.3b-1

$$g_{exterior} = \left(0.6 + \frac{d_e}{10.0}\right) g_{interior}$$

Where:
d_e = distance from exterior face of the web of exterior beam and the interior edge of curb or traffic barrier, ft.

The above equation may be used if the following condition is met:

1) $-1.0\,ft. \le d_e \le 5.5\,ft.$

LRFD C 4.6.2.2.2d

Additional Provision for Exterior Girder

An additional check is required for exterior girders because the distribution factors for girders in a multigirder cross-section was determined without consideration of diaphragms or cross-frames. The following procedure is the same as the conventional approximation for loads on piles.

$$g_{exterior} = mR$$

Where:

m = multiple presence factor as described in LRFD 3.6.1.1.2

$$R = \frac{N_L}{N_b} + \frac{X_{ext.}\sum^{N_L} e}{\sum^{N_b} x^2}$$

Where:

N_L = number of lanes investigated

N_b = number of girders

$X_{ext.}$ = horizontal distance from the center of gravity of the pattern of girders to the exterior girder, ft.

e = eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders, ft.

x = horizontal distance from the center of gravity of the pattern of girders to each girder, ft.

Skew Correction Factor

For skewed bridges there is a correction factor for shear distribution to exterior girders at the obtuse corner of the bridge.

LRFD Table 4.6.2.2.3c-1

$$\text{Skew Correction Factor} = 1 + 0.2\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}(\tan\theta)$$

Where:

K_g = longitudinal stiffness parameter = $\frac{E_S}{E_D}\left(I + Ae_g^{\,2}\right)$

E_S = modulus of elasticity of beam material, ksi

E_D = modulus of elasticity of deck material, ksi

I = moment of inertia of beam, in.^{4}

e_g = distance between the center of gravity of the basic beam and center of gravity of the deck, in.

A = area of beam, in.^{2}

The above equation may be used if the following conditions are met:

1) $3.5\,ft. \le S \le 16.0\,ft.$
2) $0° \le \theta \le 60°$
3) $20.0\,ft. \le L \le 240.0\,ft.$
4) $N_b \ge 4$

In determining the end shear in multi-beam bridges, the skew correction at the obtuse corner shall be applied to the entire span for all the beams, which is conservative for positive reaction and shear. However if uplift is a concern, uplift shall be checked without applying the Skew Correction Factor.

The Skew Correction Factor will be applied only when using the simplified equations shown above. The Skew Correction Factor will not be used with the Lever Rule or special analysis.

4.4 Dead Load Distribution

Dead load shall be applied to the following structure types as follows:

For Steel or Concrete Girder Structures
Non-composite dead loads (slab, girder) shall be distributed longitudinally to girders assuming simple supports and transversely to girders according to tributary widths.

Composite dead loads (future wearing surface, safety barrier curb) shall be distributed equally to all girders.

For Concrete Slab Structures
Entire dead load should be distributed across the full width of slab.

Longitudinally, heavier slab portions may be considered as concentrated loads in analyzing the structure.

For transverse bent design, consider the dead load reaction at the bent to be a uniform load across entire length of the transverse beam.

4.5 Longitudinal Wind Force Distribution

The longitudinal wind force as calculated in LRFD DG Sec. 1.2.2.3 shall be distributed to each bent/support for the purpose of substructure design. The force applied to each bent/support shall be calculated by using the following procedure.

The total longitudinal wind load applied to the superstructure of a continuous series of spans causes a small movement, deflecting each support by an equal amount. This relation is described in the following equation:

$$\Delta_1 = \Delta_2 = \ldots = \Delta_i = \ldots = \Delta_n$$

Where:
Δ_i = The total deflections at Bent *i*
i = Bent (support) number
n = Total number of bents (supports)

At each bent there is a force, Pi, which causes each individual deflection. The sum of the forces equals the total longitudinal wind force as shown in the following equation:

$$LW = P_1 + P_2 + \ldots + Pi + \ldots + P_n$$

Where:
LW = Total longitudinal wind load (lbs)
P_i = Longitudinal wind load to Bent *i*
i = Bent number
n = Total number of bents

When the previous two equations are combined, a percentage of the total longitudinal wind force applied at each individual bent may be found.

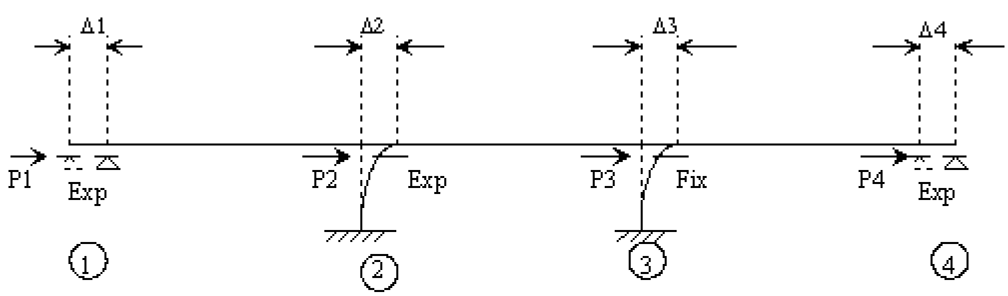

Figure 1.2.4.5.1 Support Deflections Due to Longitudinal Wind

Step 1 – Calculate Deflections at each Bent
Deflections at each bent are caused by either the elastomeric bearing pads deflecting or the column/piles deflecting.

Bearing pad deflection is calculated from the following equation (Use $\Delta Pads$ = 0 if there are no expansion pads).

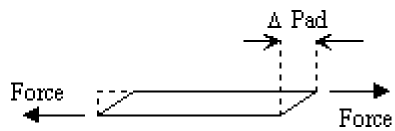

Figure 1.2.4.5.2 Deflection of Elastomeric Bearing Pad

$$\Delta Pads = \frac{P_i \times T}{L \times W \times G \times N}$$

Where:
P_i = Longitudinal force to Bent i, lbs
N = Total number of pads at Bent i
L = Length of pad, in.
W = Width of pad, in.
T = Total thickness of elastomer layers for pad, in.
G = Shear Modulus, psi

The shear modulus, G, varies with durometer, temperature, and time. To simulate this variance, the designer should run two sets of calculations, with G maximum and minimum. The values used shall be:
G_{min} = 150 psi
G_{max} = 300 psi

Column/pile deflections can be calculated from the following equation. Use $\Delta Cols$ = 0 for non-flexible bents, i.e. semi-deep abutments or non-integral end bents.

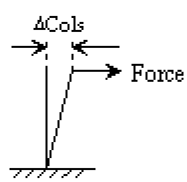

Figure 1.2.4.5.3 Column Deflection

$$\Delta Cols = \frac{P_i \times H^3}{3 \times E \times I}$$

Where:
P_i = Longitudinal force to Bent i, lb.
H = Bent height form point of fixity to top of beam, in. (*)
I = Gross moment of inertia of bent, in.^{4}, adjusted for skew (**)
E = Column modulus of elasticity, psi

* For Pile Cap Intermediate Bents or Integral End Bents, use clear height plus Equivalent Cantilever Length defined in Seismic Design Section.

** See LRFD DG Sec. 1.3 for Gross Moment of Inertia for Column & Pile Cap Bents and Resultant Moment of Inertia.

Thus, the total deflection at bent *i*, Δ_i is:
$\Delta_i = \Delta Pads + \Delta Cols$

Step 2 – Convert Loads into terms of P_1

Since:
$\Delta_1 = aP_1$
$\Delta_2 = bP_2$
$\Delta_3 = cP_3$
$\Delta_4 = dP_4$
Where:
a, b, c, & d = known values from deflection equations

And:
$\Delta_1 = \Delta_2 = \Delta_3 = \Delta_4$

Put P_i in terms of P_1:

$P_1 = P_1$
$P_2 = (a/b)P_1$
$P_3 = (a/c)P_1$
$P_4 = (a/d)P_1$

Step 3 – Find Percentage of Wind Force to Each Bent

Since:
$LW = P_1 + P_2 + P_3 + P_4$
Then:
$LW = P_1 + (a/b)P_1 + (a/c)P_1 + (a/d)P_1$
Reduces to:
$LW = [1 + (a/b) + (a/c) + (a/d)]P_1$

Thus, P_1 is found by:
$P_1 = [1/(1 + (a/b) + (a/c) + (a/d))]LW$

Once P_1 is known, the rest of the forces can then be calculated.

Note: Use the greater percentage calculated from the two different shear moduli, G, for the substructure design.

4.6 Longitudinal Temperature Force Distribution

The longitudinal temperature forces applied to a continuous series of girders causes an incremental movement which deflects the supporting columns/piles and bearing pads. The amount each component deflects is dependant on the distance from the location the movement is being investigated to the point of thermal origin. Where the point of thermal origin is the location on the bridge where there is no thermal movement.

The deflection of each bent is calculated by the following equation:

$\Delta T_i = \alpha(L_i)(T_{f,r})$

Where:

ΔT_i = the temperature movement at bent *i*

α = coefficient of thermal expansion,
= 0.000006 for concrete girders,
= 0.0000065 for steel girders

L_i = length from thermal origin to bent *i*

T_f = temperature fall as described in LRFD DG Sec. 1.2.2.4

T_r = temperature rise as described in LRFD DG Sec. 1.2.2.4

The longitudinal forces applied at each bent can be determined by the following procedure.

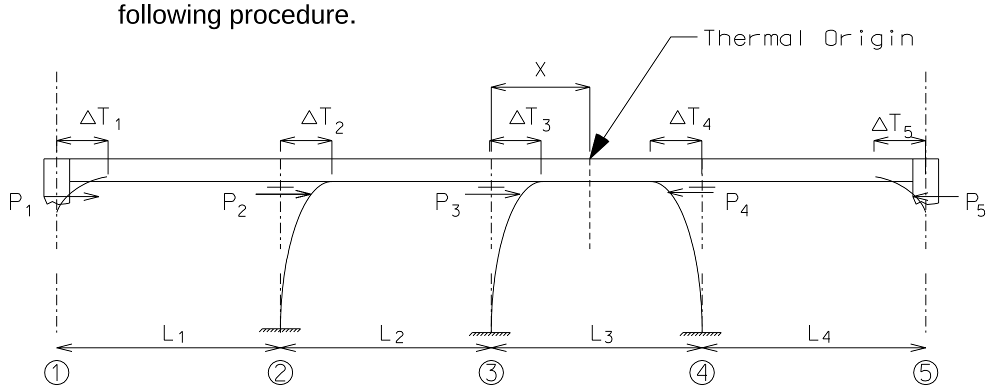

Figure 1.2.4.6.1 Support Deflections Due to Temperature Forces

Step 1 – Assume location of thermal origin

The location of the thermal origin may be assumed anywhere along the bridge. It is usually close to the middle of entire bridge length. The only care that must be taken is with the sign convention. As with the location of the thermal origin shown in Figure 1.2.4.6.1, the forces caused on the left side of the point of thermal origin must equal the forces on the right side of the thermal origin.

Thus, for Figure 1.2.4.6.1:

$P_1 + P_2 + P_3 = P_4 + P_5$

Step 2 – Calculate temperature movement at each bent in terms of distance "X"

$\Delta T_1 = (L_1 + L_2 + X)\alpha T_{f,r}$
$= a + bX$

$\Delta T_2 = (L_2 + X)\alpha T_{f,r}$

$= c + bX$
$\Delta T_3 = (X)\alpha T_{f,r}$
$= bX$ (in.)
$\Delta T_4 = (L_4 - X)\alpha T_{f,r}$
$= d - bX$
$\Delta T_5 = (L_5 + L_4 - X)\alpha T_{f,r}$
$= e - bX$

Where:
X = the temperature movement from Bent 3 to the point of thermal origin (ft.)
ΔT_i = Total deflection at Bent i (in.)
L_i = span length
i = Bent number

Step 3 – Calculate deflection computations at each bent in terms of force, P_i
Deflections at each bent are caused by either the elastomeric bearing pads deflecting or the column/piles deflecting.

Bearing pad deflection is calculated from the following equation (Use $\Delta Pads$ = 0 if there are no expansion pads).

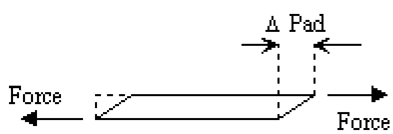

Figure 1.2.4.6.2 Deflection of Elastomeric Bearing Pad

$$\Delta Pads = \frac{P_i \times T}{L \times W \times G \times N}$$

Where:
P_i = Longitudinal force to Bent i, lbs
N = Total number of pads at Bent i
L = Length of pad, in.
W = Width of pad, in.
T = Total thickness of elastomer layers for pad, in.
G = Shear Modulus, psi

The shear modulus, G, varies with durometer, temperature, and time. For 60 Durometer pads, use G maximum associated with temperature fall, and G minimum associated with temperature rise. The values used shall be:
G_{min} = 150 psi
G_{max} = 300 psi

Column/pile deflections can be calculated from the following equation. Use $\Delta Cols$ = 0 for non-flexible bents, i.e. semi-deep abutments or non-integral end bents.

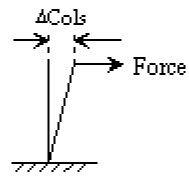

Figure 1.2.4.6.3 Column Deflection

$$\Delta Cols = \frac{P_i \times H^3}{3 \times E \times I}$$

Where:
P_i = Longitudinal force to Bent i, lb.
H = Bent height form point of fixity to top of beam, in. (*)
I = Gross moment of inertia of bent, in.^{4}, adjusted for skew (**)
E = Column modulus of elasticity, psi

* For Pile Cap Intermediate Bents or Integral End Bents, use clear height plus Equivalent Cantilever Length defined in Seismic Design Section.

** See LRFD DG Sec. 1.3 for Gross Moment of Inertia for Column & Pile Cap Bents and Resultant Moment of Inertia.

Thus, the total deflection at bent *i*, Δ_i is:
$\Delta T_i = \Delta Pads + \Delta Cols$
Or for each individual bent:
$\Delta T_1 = fP_1$
$\Delta T_2 = gP_2$
$\Delta T_3 = hP_3$
$\Delta T_4 = jP_4$
$\Delta T_5 = kP_5$
Where:
f, g, h, j, & k = known values from deflection equations

Step 4 – Set thermal deflection in terms of X equal to the deflection in terms of P_i for the corresponding bent numbers
Once the terms are combined in one equation, solve each bent equation for the corresponding bent force P_i.
Where:
$fP_1 = a + bX \Rightarrow P_1 = (a + bX)/f$
$gP_2 = c + bX \Rightarrow P_2 = (c + bX)/g$
$hP_3 = bX \Rightarrow P_3 = (bX)/h$
$jP_4 = d - bX \Rightarrow P_4 = (d - bX)/j$
$kP_5 = e - bX \Rightarrow P_5 = (e - bX)/k$

Step 5 - Solve for X
Since:
$P_1 + P_2 + P_3 = P_4 + P_5$

Substitue equations from Step 4 into the above equation and solve for X

Step 6 – Calculate forces

Once the variable X is calculated, the values for each temperature force applied to the bents can be found. Two runs shall be done to account for for both "temperature rise" and "temperature fall". The maximum load found between the two cases shall be used in design.